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Canadian highway bridge evaluation: load and resistance factors

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Consistent load and resistance factors are developed for a range of target values of the reliability index, β , following first-order second-moment analysis techniques for use in the evaluation of highway bridges. Dead load factors are established for steel girders, concrete girders, concrete bridge decks, and wearing surfaces, taking into account the statistical variations of weights and the range of load fractions as determined from field measurements. Live load factors are established for four categories of live loads: NP — non-permit traffic that are permitted by legislation; PM — permit, multiple trip, bulk haul, divisible loads; PS — permit, single trip, unsupervised, mixed with non-permit traffic; and PC — permit, controlled, supervised extremely heavy loads with escort. These live load factors are based on field surveys of truck weights, in Alberta and elsewhere. The event curves for NP, PS, and PM traffic have been used to determine the maximum annual truck, as the period of evaluation was chosen as 1 year based on a life-safety criterion related to the consequences of failure. Because PC traffic is so rare, it was dealt with on an event basis. Impact data of others were analyzed to determine the appropriate bias coefficients and coefficients of variation. Uncertainties in the transverse distribution of both dead and live loads were also considered.

Resistance factors are based on statistical data reported in the literature and take into account the variation in material properties, member size, and the resistance formulations.

Key words: dead and live load factors, resistance factors, impact, maximum annual, traffic categories, transverse distribution, weight fractions.

Des coefficients uniformes de pondération des surcharges et de la résistance ont été développés pour une gamme de valeurs cibles de l'indice de fiabilité β suivant des techniques d'analyse du second moment de premier ordre afin d'évaluer les ponts-routes. Des coefficients de pondération des charges permanentes ont été établis pour les poutres d'acier, les poutres de béton, les tabliers en béton et les couches d'usure en prenant en considération les variations statistiques de poids et la gamme de fractions de surcharges déterminées à partir de mesure in situ. Des coefficients de pondération des charges permanentes ont été établis pour quatre catégories de charge, soit la circulation NP (véhicules sans permis), la circulation PM (véhicules avec permis à déplacements multiples, transport en vrac, charges divisibles), circulation PS (véhicules avec permis à déplacement unique, sans surveillance et combinés à des véhicules sans permis) et circulation PC (véhicules avec permis, contrôlés et avec surveillance, charges très lourdes avec escorte). Ces coefficients sont basés sur des études in situ de poids de camions en Alberta et ailleurs. Les courbes d'événement pour les circulations NP, PS et PM ont été utilisées afin de déterminer la charge annuelle maximale, car la période d'évaluation choisie est d'une durée de un an basée sur un critère de sécurité relié aux conséquences d'une défaillance. Parce que le volume de circulation PC est infime, il a été pris en considération sur une base événementielle. Les données de choc des autres ont été analysées afin de déterminer les coefficients d'écart systématique et de variation. Des incertitudes dans la répartition transversale des charges permanentes et temporaires ont également été prises en considération.

Les coefficients de pondération de la résistance sont basés sur les paramètres statistiques mentionnés dans les ouvrages de référence et prennent en considération la variation des propriétés matérielles, la dimension des éléments et les formules de résistance.

Mots clés : coefficients de pondération des charges permanentes et temporaires, coefficients de pondération de la résistance, choc, annuel maximal, catégories de circulation, répartition transversale, fractions de poids.

[Traduit par la rédaction]

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1. Introduction

Limit states design is conceptually suited to the development of live load rating factors for the evaluation of bridges

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and the components of bridges for various categories of highway traffic loading. The statistical variations of the loads and resistances, as measured by their bias coefficients and coefficients of variation, are considered, using first-order second-moment analyses, in the development of the load and resistance factors. These factors impinge directly on the live load rating factor. The bridge component with the least live load rating factor is critical. Furthermore, in developing the load and resistance factors, different values

of the reliability index, β , can be used, as recommended by Allen (1992), based on a life-safety criterion related to the consequences of failure of the component.

The objective of this paper is to determine the load and resistance factors for four categories of truck traffic, for a variety of bridge components, for different methods of transverse load distribution, and for a range of β values.

The methodology, following first-order second-moment analysis procedures, is first presented. Because the resistance and load factors depend on the fraction of the total load effect that each of the dead and live loads (including impact) contributes, load effect fractions for a sample of bridges, considered to be reasonably representative, are presented followed by statistical data on the dead loads, live loads, impact, uncertainties in the transverse distribution of live and dead loads, and the resistances of various components. The data on load effect fractions, dead loads, and live loads are based on field surveys conducted for this investigation. A matrix of load and resistance factors are then determined for the case when the dead and live loads both load the structural component and for the case when the dead loads are counteractive.

2. Methodology

2.1. General case

The live load rating factor, LLRF, as described by Buckland and Bartlett (1992) is given by

$$[1] \quad \text{LLRF} = \frac{\phi R - \Sigma \alpha_D D}{\alpha_L L (1 + \alpha_I I)}$$

where R , D , L , and I are nominal values of the resistances and load effects.

Equation [1] is simply a rearrangement of

$$[1a] \quad \phi R = \Sigma \alpha_D D + \text{LLRF} \alpha_L L (1 + \alpha_I I)$$

from which it is seen that the LLRF is the multiple of the prescribed factored live load, including impact, that the existing bridge or component, with an established factored resistance, ϕR , can carry. It is greater than 1, equal to 1, or less than 1 when the factored resistance is respectively greater than, equal to, or less than the effect of factored loads. Depending, as it does, on the difference between the factored dead load effect, it is sensitive to the resistance and dead load factors and also to the live load factor which therefore deserve special attention.

Load and resistance factors (α and ϕ) are derived starting with (Nowak and Lind 1979a)

$$[2] \quad \bar{R} = \bar{S} \exp(\beta \sqrt{V_S^2 + V_R^2})$$

where $\bar{R}$ and $\bar{S}$ are the mean values of resistances and load effects, and V_R and V_S are the associated coefficients of variation. (Nowak and Lind (1979b) discuss calibration procedures more generally.)

The limit states design inequality is

$$[3] \quad \phi R \geq \Sigma \alpha_S S_i$$

and noting that

$$[4] \quad \bar{R} = \delta_R R$$

substituting [3] and [4] in [2] gives, when using the least value for ϕR from [3],

$$[5] \quad \Sigma \alpha_S S_i = \frac{\phi \bar{S}}{\delta_R} \exp(\beta \sqrt{V_S^2 + V_R^2})$$

The total nominal load is the sum of the loads

$$[6] \quad T = D_1 + D_2 + D_3 + D_4 + L + LI$$

The nominal load effect, taking into account the uncertainty in the analysis of the loads, E , is

$$[7] \quad S = ET = E(D_1 + D_2 + D_3 + D_4 + L + LI)$$

The factored load effect, based on the same nominal loads, is

$$[8] \quad \Sigma \alpha_S S_i = E[\alpha_{D_1} D_1 + \alpha_{D_2} D_2 + \alpha_{D_3} D_3 + \alpha_{D_4} D_4 + \alpha_L L (1 + \alpha_I I)]$$

where I is the impact expressed as a fraction of the live load; and the effective load factor for impact is therefore the product $\alpha_L \alpha_I$. The mean load effect, $\bar{S}$, can be written as

$$[9] \quad \bar{S} = \bar{ET} = \bar{E}[\bar{D}_1 + \bar{D}_2 + \bar{D}_3 + \bar{D}_4 + \bar{L}(1 + \bar{I})]$$

Each of the mean values of load can be expressed, for example, as

$$[10] \quad \bar{D}_1 = \frac{\bar{D}_1}{D_1} D_1 = \delta_{D_1} D_1$$

Incorporating [8], [9], and [10] in [5] and dividing by $S = ET$ gives

$$[11] \quad [\alpha_{D_1} D_1 + \alpha_{D_2} D_2 + \alpha_{D_3} D_3 + \alpha_{D_4} D_4 + \alpha_L L (1 + \alpha_I I)] / T = \frac{\phi \bar{E}}{\delta_R ET} [\delta_{D_1} D_1 + \delta_{D_2} D_2 + \delta_{D_3} D_3 + \delta_{D_4} D_4 + \delta_L L (1 + \delta_I I)] \exp(\beta \sqrt{V_S^2 + V_R^2})$$

For dead loads, the mean value of the uncertainty in analysis is taken to be the nominal value because statics will always be satisfied. Therefore, the bias coefficient, $\delta_E = \bar{E}/E$, equals 1.0. For live loads, the mean value of the uncertainty in analysis may differ from 1 and therefore must be taken into account. Thus

$$[12] \quad \frac{\bar{E}}{E} \delta_L L = \delta_{EL} \delta_L L$$

This factor also applies to the impact fraction derived from the live load, and therefore [11] becomes

$$[13] \quad [\alpha_{D_1} D_1 + \alpha_{D_2} D_2 + \alpha_{D_3} D_3 + \alpha_{D_4} D_4 + \alpha_L L (1 + \delta_I I)] / T = \frac{\phi R}{\delta_R RT} \exp(\beta \sqrt{V_S^2 + V_R^2}) \times [\delta_{D_1} D_1 + \delta_{D_2} D_2 + \delta_{D_3} D_3 + \delta_{D_4} D_4 + \delta_{EL} \delta_L L (1 + \delta_I I)]$$

In deriving [13], each load effect was divided by the total load effect, S , and therefore becomes a load effect fraction expressed as a decimal. Therefore, to establish consistent load and resistance factors from [13], not only are the statistical data in the form of bias coefficients and coefficients of variation needed for each of the load components and for the particular resistance in question but also the

decimal fractions of the load effects for a representative group of bridges or bridge members. (An alternate approach to the one used here would be to establish the bias coefficients and coefficients of variation for each of the load effect fractions.)

2.2. Coefficient of variation of the effect of loads, V_S

The coefficient of variation for the effect of loads can be determined simply as the square root of the sum of the squares of the coefficients of variation of the load components, provided that these components are independent. However, when some of the load effects are interrelated, as is the case for live load and impact, or when they are not independent as may be the case for the analysis for transverse distribution of loads, this should be taken into consideration.

If the coefficient of variation of the analysis for transverse distribution of loads, V_E , is the same for all loads, then the coefficient of variation for load effects is given by

$$[14] \quad V_S^2 = V_T^2 + V_E^2$$

in which

$$[15] \quad V_T = \frac{\sigma_T}{\delta_T T}$$

$$[16] \quad \sigma_T^2 = \Sigma(V_j \delta_j D_j)^2 + (V_{L_i} \delta_{L_i} L_i)^2$$

The effect of live load and impact can be written as

$$[17a] \quad L_i = L I_f$$

or

$$[17b] \quad L_i = L(1 + I)$$

where I_f is the dynamic load factor and I is the dynamic load fraction. Because the live load and impact are generally considered to be independent, then

$$[18] \quad V_{L_i}^2 = V_L^2 + V_I^2$$

But the variance of the impact factor is the same as the variance of the impact fraction, because shifting a distribution curve by a constant value (here 1) does not change the variance, and therefore

$$[19] \quad \sigma_{I_f}^2 = \sigma_I^2$$

to give

$$[20] \quad V_{I_f} \bar{I}_f = V_I \bar{I} = V_I \delta_I I$$

or

$$[21] \quad V_{I_f} = \frac{V_I \bar{I}}{\bar{I}_f} = \frac{V_I \delta_I I}{(1 + \delta_I I)}$$

Thus,

$$[22] \quad V_{L_i}^2 = V_L^2 + \left(\frac{V_I \delta_I I}{(1 + \delta_I I)} \right)^2$$

It remains to take into account the non-constant value for the coefficient of variation of the lateral load distribution for the different loads.

Let δ_{ED_1} , δ_{ED_2} , δ_{ED_3} , δ_{ED_4} , and δ_{EL} be the bias coefficients of the analysis for the lateral load distribution for dead loads 1, 2, 3, 4, and live load (including impact) respectively. The associated coefficients of variation are V_{ED_1} , V_{ED_2} , V_{ED_3} , V_{ED_4} , and V_{EL} . Suppose that, of these, V_{ED_1} is the minimum. We subtract this minimum value using the root-mean-square method from all the others and reintroduce it again applied to the total load effect. Thus,

$$[23a] \quad V_{ED_2}'^2 = V_{ED_2}^2 + V_{ED_1}^2$$

$$[23b] \quad V_{ED_3}'^2 = V_{ED_3}^2 + V_{ED_1}^2$$

$$[23c] \quad V_{ED_4}'^2 = V_{ED_4}^2 + V_{ED_1}^2$$

$$[23d] \quad V_{EL}'^2 = V_{EL}^2 + V_{ED_1}^2$$

The variance for the total load effect, excluding the contribution of V_{ED_1} , is then

$$[24] \quad \sigma_S'^2 = (V_{ED_1}'^2 + V_{D_1}^2)(\delta_{D_1} \delta_{ED_1} D_1)^2 + (V_{ED_2}'^2 + V_{D_2}^2)(\delta_{D_2} \delta_{ED_2} D_2)^2 + (V_{ED_3}'^2 + V_{D_3}^2)(\delta_{D_3} \delta_{ED_3} D_3)^2 + (V_{ED_4}'^2 + V_{D_4}^2)(\delta_{D_4} \delta_{ED_4} D_4)^2 + (V_{EL}'^2 + V_L^2)(\delta_L \delta_{EL} L(1 + \delta_I I))^2$$

and

$$[25] \quad V_S'^2 = \frac{\sigma_S'^2}{\delta_S^2}$$

where

$$[26] \quad \delta_S = \delta_{D_1} \delta_{ED_1} D_1 + \delta_{D_2} \delta_{ED_2} D_2 + \delta_{D_3} \delta_{ED_3} D_3 + \delta_{D_4} \delta_{ED_4} D_4 + \delta_L \delta_{EL} L(1 + \delta_I I)$$

now

$$[27] \quad V_S^2 = V_S'^2 + V_{ED_1}^2$$

and

$$[28] \quad V_S = \sqrt{V_S'^2 + V_{ED_1}^2}$$

2.3. Counteractive dead loads

Under certain circumstances, particularly with continuous bridges, the dead load effects at a certain location of the bridge may counteract the effects due to live load. Under these circumstances, the dead loads, in effect, resist the live load effects and are properly considered to act with the resistances. Equation [13] is reformulated as

$$[29] \quad \alpha_L L(1 + \alpha_I I)/T = \frac{(\phi R + \alpha_{D_1} D_1 + \alpha_{D_2} D_2 + \alpha_{D_3} D_3 + \alpha_{D_4} D_4)}{(\delta_R R + \delta_{D_1} D_1 + \delta_{D_2} D_2 + \delta_{D_3} D_3 + \delta_{D_4} D_4)T} \times \exp(\beta \sqrt{V_S'^2 + V_R'^2}) [\delta_{EL} \delta_L L(1 + \delta_I I)]$$

TABLE 1. Nominal load effect fractions

General information	Load effect	D_1/T or D_2/T	D_3/T	D_4/T	L/T	I/T
Alberta						
1. Precast voided section, 11 m span	Midspan moment	0.258	0.0	0.025	0.552	0.165
2. Precast channel, 8.5 m span	Midspan moment	0.222	0.0	0.017	0.585	0.176
3. Precast voided section, 10 m span	Midspan moment	0.362	0.0	0.022	0.474	0.142
4. Precast channel with bulbs, 16.8 m span	Midspan moment	0.533	0.0	0.022	0.342	0.103
5. Precast channel with bulbs, 34 m span	Midspan moment	0.566	0.0	0.090	0.284	0.060
6. Precast deck bulb tees, 10 m span	Midspan moment	0.107	0.0	0.018	0.673	0.202
7. Precast deck bulb tees, 20 m span	Midspan moment	0.144	0.0	0.024	0.616	0.216
8. Precast deck bulb tees, 30 m span	Midspan moment	0.188	0.0	0.032	0.607	0.173
9. Precast deck bulb tees	Midspan moment	0.282	0.0	0.048	0.541	0.129
10. Rolled steel beam	Midspan moment	0.072	0.213	0.034	0.500	0.181
11. Rolled steel beam, 27.4 m span	Midspan moment	0.067	0.275	0.046	0.495	0.117
12. Rolled steel beam, 30.5 m span	Midspan moment	0.069	0.288	0.072	0.469	0.102
13. Rolled steel beam, 33.5 m span	Midspan moment	0.087	0.258	0.062	0.468	0.124
14. Welded plate girder, 75 m span	Midspan moment	0.086	0.260	0.024	0.546	0.084
15. Welded plate girder, 92 m span	Midspan moment	0.137	0.265	0.057	0.478	0.062
Saskatchewan						
16. Steel plate girder, simple supports, 165 ft span	Moment	0.133	0.325	0.187	0.253	0.101
17. Steel plate girder, 165 ft span	Shear	0.092	0.223	0.125	0.399	0.161
18. Steel plate girder, continuous 211 ft span	Moment	0.146	0.262	0.185	0.292	0.115
19. Steel plate girder, continuous 211 ft span	Shear	0.093	0.169	0.119	0.443	0.176
20. Concrete T-beam, 56 ft span	Shear	0.481	0.0	0.065	0.325	0.130
21. Concrete T-beam, 56 ft span	Moment	0.373	0.0	0.049	0.413	0.165
22. Precast concrete, continuous 110 ft span	Moment	0.574	0.0	0.052	0.267	0.107
23. Precast concrete, 110 ft span	Shear	0.385	0.0	0.048	0.404	0.163
24. AASHTO prestressed I-girder, 53 ft span continuous	Moment	0.166	0.306	0.024	0.360	0.144
25. AASHTO prestressed I-girder, 53 ft span	Shear	0.092	0.161	0.057	0.494	0.195
26. Steel truss, upper chord 250 ft span	Compression	0.787	0.0	0.0	0.154	0.059
27. Steel truss, stringers, 16 ft span	Shear	0.018	0.018	0.0	0.691	0.273
28. Steel girder, continuous 50 ft span	Moment	0.040	0.226	0.048	0.490	0.196
29. Steel girder, 50 ft span	Shear	0.028	0.155	0.035	0.556	0.226
30. Steel girder, simply supported 45 ft span	Moment	0.036	0.254	0.046	0.475	0.189
31. Steel girder, simply supported 45 ft span	Moment	0.019	0.152	0.029	0.571	0.229
32. Prestressed box, simply supported 9 m span	Moment	0.209	0.0	0.052	0.569	0.170
Ontario						
33. Plate girder OHBDC, 24 m span	Moment	0.044	0.220	0.080	0.499	0.157
34. Plate girder OHBDC, 30 m span	Moment	0.061	0.298	0.088	0.379	0.174
35. Plate girder OHBDC, 42 m span	Moment	0.089	0.314	0.099	0.350	0.146
36. Box girder OHBDC, 45 m	Moment	0.094	0.356	0.112	0.314	0.124
37. Box girder, AASHTO-LF 39 m	Moment	0.110	0.425	0.126	0.282	0.056
38. Box girder, 6 span, continuous, OHBDC	Positive moment	0.032	0.143	0.042	0.540	0.243
39. Box girder, 2 span, continuous, AASHTO	Positive moment	0.071	0.425	0.109	0.338	0.057
40. Plate girder, OHBDC	Reaction	0.132	0.384	0.096	0.321	0.067
41. Plate girder, AASHTO	Reaction	0.134	0.334	0.156	0.329	0.046

NOTE: 1 ft = 0.3048 m.

where V_R is computed from the sum of the variances of the resistance and the dead load effects.

Equation [29] is written for the situation when all dead loads are counteractive. When some of the dead loads are not counteractive, they appear on the left side of the equation as in [13]. The computer program was written to put active (positive) dead load effects on the left side and counteractive (negative) dead load effects on the right.

3. Load effect fractions

Table 1 gives the load effect fractions for a total of 41 different load effects collected from the provinces of Alberta, Saskatchewan, and Ontario for a variety of bridges

and bridge members. In Table 2, similar data are presented for 15 different load effects, collected in Alberta, for cases when some or all of the dead load effect fractions counteract live load effects.

The ratios are for various bridge members and types — steel and concrete members in flexure, shear, and compression. In some of these, the dead load is dominant; in others, the live load is dominant.

4. Dead load data

The data presented here together with sources of information are given in Table 3.

TABLE 2. Nominal load effect fractions with dead load counteractive

General information	Load effect	D_1/T or D_2/T	D_3/T	D_4/T	L/T	I/T
Alberta						
1. Cast-in-place concrete girders 17.7-25.0-25.0-17.7 m continuous spans	Moment	0.0	-0.01	0.0	0.72	0.29
2. Cast-in-place concrete girders 17.7-25.0-25.0-17.7 m continuous spans	Moment	0.0	-1.38	-0.18	1.83	0.73
3. Cast-in-place concrete girders 27.4-37.8-37.8-27.4 m continuous spans	Moment	0.0	0.01	-0.14	0.82	0.33
4. Cast-in-place concrete girders 27.4-37.8-37.8-27.4 m continuous spans	Moment	0.0	0.0	-0.06	0.81	0.25
5. Cast-in-place concrete girders 27.4-37.8-37.8-27.4 m continuous spans	Moment	0.0	-0.20	0.0	0.92	0.28
6. Cast-in-place concrete girders 27.4-37.8-37.8-27.4 m continuous spans	Moment	0.0	0.01	-0.12	0.86	0.27
7. Prestressed concrete girders 21.3-41.6-41.8-41.6-21.3 m continuous spans	Moment	-0.88	-0.54	0.09	1.69	0.64
8. Prestressed concrete girders 21.3-41.6-41.8-41.6-21.3 m continuous spans	Moment	-0.36	-0.17	0.10	1.04	0.39
9. Prestressed concrete girders 21.3-41.6-41.8-41.6-21.3 m continuous spans	Moment	-2.74	-1.92	0.53	3.91	1.22
10. Steel girders 19.5-24.4-19.5 m continuous spans	Moment	-0.04	-0.12	-0.03	0.92	0.27
11. Steel girders 19.5-24.4-19.5 m continuous spans	Moment	-0.02	-0.06	-0.01	0.82	0.25
12. Steel girders 19.5-24.4-19.5 m continuous spans	Moment	-0.43	-1.51	-0.34	2.52	0.76
13. Steel girders 39.6-50.0-50.0-39.6 m continuous spans	Moment	-0.02	-0.07	0.03	0.87	0.25
14. Steel girders 39.6-50.0-50.0-39.6 m continuous spans	Moment	-0.13	-0.40	-0.14	1.29	0.38
15. Steel girders 39.6-50.0-50.0-39.6 m continuous spans	Moment	-0.34	-1.04	-0.38	2.12	0.64

4.1. Steel girders, D_1

The data for steel members are based on Kennedy and Baker (1984) who gave $\delta = 1.0045$ and $V = 0.0224$ for unpainted steel. It is customary for the designer to make an allowance in his calculations for the weight of miscellaneous steel in the structure that appears in the form of bolt heads, stiffeners, diagonal, and other bracing members. For box girders a common allowance is 8%, and for plate girders the allowance can be as much as 14%. It is also common practice for the designer to check his final design against this allowance. Therefore, on average, this extra weight would be part of the nominal weight. It is unlikely that the designer would modify his design if his allowance for miscellaneous steel is too high. On the other hand, if his allowance

is marginally too low, he is also unlikely to modify his design. However, the nominal mean value before he would modify his design is likely to be less than the weight plus the allowance rather than more than the weight plus the allowance. Let us neglect, as being conservative, this change in δ . Let us assume that an allowance in the weight of 12% for miscellaneous steel has been made and that the designer accepts a deviation of one third of this, or 4%. Assuming this corresponds to 2 standard deviations, the coefficient of variation is $0.04/2 = 0.02$. Hence, V_D becomes $(0.0224^2 + 0.02^2)^{1/2} = 0.03$.

Painting and repainting, if done at all, follows blast cleaning. The paint film (two-coat system) is not likely to exceed 11 mils and is generally not taken into account by the

TABLE 3. Dead load data

Category	Source*	δ	V
D_1 — steel girders	Kennedy and Baker (1984)		
Unpainted		1.0045	0.030
Painted		1.0117	0.030
D_2 — concrete girders	Hamblin, ² Ramsay, ³	1.0436	0.0359
cast-in-place and precast	Ellingwood <i>et al.</i> (1980)		
D_3 — concrete bridge decks	Ramsay, ³ Peng, ⁴ Lester ⁵	1.068	0.1399
D_4 — wearing surfaces	Hamblin, ⁶	1.437	0.5316
	Grouni and Nowak (1984), Lester ⁵		

*Refer to footnotes in text.

designer. This mass is estimated as 0.905 kg/m^2 ; and for the deep sections used for bridges, the fraction of total weight for bridge members is estimated to be 0.0071. Therefore, δ_D for painted steel becomes 1.0045×1.0071 .

4.2. Concrete girders, D_2

The statistical data are based on information supplied by Hamblin,² Ramsay,³ and Ellingwood *et al.* (1980). Ellingwood *et al.* (1980) report that concrete densities vary from 2190 to 2480 kg/m^3 and that this range represents 4 standard deviations to give a mean value of 2335 kg/m^3 and a coefficient of variation of 0.0310. Bridge girders are likely to have about $3\% \pm 1\%$ reinforcing steel of 7850 kg/m^3 , thus increasing the mean density for a reinforced concrete girder to 2500 kg/m^3 , or 1.042 times the nominal design value of 2400 kg/m^3 . Assuming again that the variation in reinforcement represents 2 standard deviations on either side of the mean, this gives an additional coefficient of variation in density of 0.0109. The variation in girder dimensions from a total sample of 683 measurements (Hamblin,² Ramsay³) gives a δ of 1.0015 and a V of 0.0144. Combining these data gives $\delta = 1.0436$ and $V = 0.0359$. Precast girders are assumed to have the same variation as cast-in-place girders. Grouni and Nowak (1984) give $\delta = 1.03$ and $V = 0.04$ for precast concrete components, which agrees well with the values calculated here.

4.3. Concrete bridge decks, D_3

Concrete densities are again based on the data of Ellingwood *et al.* (1980). The deck steel percentage is taken, on the average, to be 1.5% to give an overall average density of 2418 kg/m^3 or a δ of 1.0075 based on the nominal value of 2400 kg/m^3 . The variation in steel percentage does not appreciably affect the coefficient of variation of concrete density. The data from Ramsay³ on one deck, from Peng⁴

on five decks, and from Lester⁵ on seven decks gave a δ of 1.060 and a V of 0.1364 for bridge deck thicknesses. Combining these data gives a δ of 1.068 and a V of 0.1399.

4.4. Wearing surfaces, D_4

Unless the evaluator measures the actual thickness of asphalt or other wearing surface, he is obliged to use the data based on the general population. The data from Hamblin⁶ gave a δ of 1.56 and a coefficient of variation of 0.520. Grouni and Nowak (1984) report $\delta = 1.05$ and $V = 0.25$, while Lester reports $\delta = 1.702$ and $V = 0.522$. Combining these three populations with equal weighting gives $\delta = 1.437$ and $V = 0.5316$.

5. Live load data

Live loads are divided into four categories:

- NP — non-permit, permitted by legislation;
- PM — permit, multiple trip, bulk haul, divisible load;
- PS — permit, single trip, unsupervised, mixing with the general traffic;
- PC — permit, controlled, supervised.

5.1. NP traffic

A total of 151 765 northbound trucks on Alberta Highway 2, section 30, in a period of 44 weeks extending from February 1, 1983, to December 5, 1983, were identified and weighed as to truck type by a weigh-in-motion scale (Christison 1985). Normal distribution curves of arbitrary sample size of 5000 to 30 000 vehicles were fitted to the upper tail to provide heavy truck event distribution curves. Each of these truck event curves was then used to derive a maximum truck curve for the evaluation period from

$$[30] \quad p_{HM} = 1 - (1 - p_H)^M$$

where p_H is the probability of exceeding a weight H in the truck event distribution; p_{HM} is the probability of exceeding

²Hamblin, L.J. 1987. Survey of concrete girders in Saskatchewan. Saskatchewan Highways and Transportation, Regina, Sask. Private communication.

³Ramsay, R.J. 1987. Survey of concrete girder dimensions in Alberta. Alberta Transportation and Utilities, Edmonton, Alta. Private communication.

⁴Peng, S.C. 1987. Survey of deck thicknesses of five bridges in Yukon. Public Works Canada, Ottawa, Ont. Private communication.

⁵Lester, P. 1988. Survey of deck and wearing surface thickness of eight bridges in Newfoundland. Government of Newfoundland and Labrador, Department of Transportation, St. John's, Nfld. Private communication.

⁶Hamblin, L.J. 1987. Survey of asphalt wearing surfaces in Saskatchewan. Saskatchewan Highways and Transportation, Regina, Sask. Private communication.

a weight H in the time period of the evaluation; and M is the number of heavy trucks in the time period of the evaluation.

For the event distribution curve, the probability p_H is the probability that the next truck on the bridge exceeds a weight of H , while for the time period of evaluation, the probability P_{HM} is the probability that a truck within the time period exceeds a weight of H .

The maximum truck curve was based on an evaluation period of 1 year for non-permit (NP) traffic, as this period relates well to human safety and because some traffic loads may be seasonal as are environmental loads. As well, using a 1-year period avoids the dangers of changing conditions, bridge deterioration, and extrapolations for traffic load growth and traffic volume growth. The safety index, β , however, must be established for the 1-year period as compared to that used for a 50-year design life (see Allen 1992).

Line A in Fig. 1 is fitted to the data for a heavy truck sample of 5000 trucks taken from the upper end of the total sample. The mean value of the heavy truck is 46.5 t with a standard deviation of 7.92 t and a coefficient of variation of 0.170. As this sample of 5000 trucks was for a period of 308 days, the number of trucks in the time period of 1 year would be $(365/308) \times 5000 = 5925$. Allowing for 24% of the trucks that missed the scale (Christison 1985), the number per year becomes $5925/0.76 = 7796$.

The data corresponding to the maximum annual truck curve are determined by first establishing the probability of exceedance, p_H , for any gross vehicle weight from curve A. For example, for 75.0 t, 3.60 standard deviations from the mean, $p_H = 0.15911 \times 10^{-3}$, as may be determined from normal curve area tables. This weight, from [30], therefore corresponds to

$$[30b] \quad P_{HM} = 1 - (1 - 0.15911 \times 10^{-3})^{7796} = 0.7107$$

By repeating this procedure a number of times, sufficient points are established to allow the maximum annual truck curve B to be drawn.

In Fig. 1, it is seen that line A and, more particularly, line B deviate from the plotted data for higher probabilities of exceedance. This indicates that the distributions are not normal distributions (that for the annual event (B) is actually an extreme value distribution). However, by fitting the straight line of a normal distribution to the critical upper tail of the data, the curves, whatever they may be, are matched here and errors are minimized. If a steeper line had been used, while the mean value would have been increased (unconservative effect), the coefficient of variation would have decreased (conservative effect).

The resulting maximum annual truck curve is insensitive to the arbitrary size selected for the heavy truck event curve. For example, for initial populations of 5000, 11 325, and 30 000, the bias coefficients (expressed as a ratio to the legal maximum truck weight of 53.5 t) were 1.36, 1.37, and 1.36, respectively, and the coefficients of variation were 0.0686, 0.0658, and 0.0710, respectively. This is true because it is the same data base at the upper end of the sample that is being used. Differences are attributed to curve fitting.

It is noted that the bias coefficient of the maximum annual truck, 1.36, is greater than that of the truck event curve, $46.5/53.5 = 0.869$, but that the coefficient of variation is reduced to 0.0686 from 0.170. These two effects therefore compensate each other. It follows that if a longer evaluation

period were used, the mean truck weight would increase further, but the coefficient of variation would be reduced. In Fig. 1, for longer evaluation periods, the derived curves, such as B, shift further to the right and have steeper slopes. For the annual curve, the relatively large value of $\delta = 1.36$ is related to the low reference value of 53.5 t.

5.2. PM traffic

Moroz⁷ provided data on 242 multiple trip events with weights of up to 58.5 t. These vehicles had a very narrow weight distribution, with a mean value of the measured weight to permit weight of 1.0069 and a coefficient of variation of 0.0155. Hamblin⁸ provided data for 32 PM trucks, with $\delta = 0.995$ and $V = 0.0161$. The data supplied by Buckland & Taylor Ltd.⁹ on 251 ore haul vehicles in the Yukon gave a δ of 1.0076 and a V of 0.00641. Combining these data, assigning equal weight to each of the three samples, for a total sample of 525 trucks results in a δ of 1.0032 and a V of 0.0146. The 1-year maximum truck curves, based on a truck population of 43 800 trucks/annum on the route (120 trucks/day) has a δ of 1.063 and a V of 0.0094. Because of the narrowness of the distribution, the maximum annual PM curve is not sensitive to the annual number of trucks.

5.3. PS traffic

Moroz¹⁰ provided data on a sample of 109 truck events based on the data for 1 week obtained in October 1987 from three different weigh stations. Weights of up to 183.8 t were recorded. The mean ratio of the measured weight to that estimated by those seeking the permits was 0.9994, with a coefficient of variation of 0.0648. Additional data collected in 1988 from Moroz¹¹ for 102 trucks gave $\delta = 1.00$ and $V = 0.045$, while the data from Hamblin¹² for 12 trucks gave $\delta = 0.973$ and $V = 0.071$. Combining all these data, giving equal weight to each of the 223 trucks, results in truck event statistics of $\delta = 0.9984$ and $V = 0.0569$. When extrapolated to the maximum annual truck, a mean value of 1.168 and a coefficient of variation of 0.0295 are obtained for an annual population of 1614 trucks.

5.4. PC traffic

This traffic is relatively rare but heavy. Moroz¹³ pro-

⁷Moroz, A.W. 1987. Survey of permit multi-trip vehicles in Alberta, 1987. Alberta Transportation and Utilities, Edmonton, Alta. Private communication.

⁸Hamblin, L.J. 1988. Survey of permit multi-trip vehicles in Saskatchewan, 1988. Saskatchewan Highways and Transportation, Regina, Sask. November. Private communication.

⁹Buckland & Taylor Ltd. 1988. Survey of permit multi-trip vehicles in the Yukon. Vancouver, B.C. April. Private communication.

¹⁰Moroz, A.W. 1987. Survey of permit single trip vehicles in Alberta, 1987. Alberta Transportation and Utilities, Edmonton, Alta. Private communication.

¹¹Moroz, A.W. 1988. Survey of permit single trip vehicles in Alberta, 1988. Alberta Transportation and Utilities, Edmonton, Alta. Private communication.

¹²Hamblin, L.J. 1988. Survey of permit single trip vehicles in Saskatchewan, 1988. Saskatchewan Highways and Transportation, Regina, Sask. Private communication.

¹³Moroz, A.W. 1987. Survey of permit controlled vehicles in Alberta. Alberta Transportation and Utilities, Edmonton, Alta. Private communication.

TABLE 4. Live load data

Category	Source*	δ	V
NP — general highway loading, maximum annual truck	Christison (1985)	1.36	0.0686
PM — permit, multiple trip, maximum annual truck	Moroz, ⁷ Hamblin, ⁸ Buckland & Taylor Ltd. ⁹	1.063	0.0094
PS — permit, single trip, maximum annual truck	Moroz, ^{10,11} Hamblin ¹²	1.168	0.0295
PC — permit, controlled, truck event	Moroz ¹³	1.002	0.385

*Refer to footnotes in text.

vided a sample of six trucks only, which showed the distribution to be extremely narrow with a mean weight very close to the estimated value. This traffic is dealt with on an "event basis." The δ and V are 1.002 and 0.0385 respectively.

5.5. Traffic on short spans

For short spans (short influence lines), the variability of the axle weights is of more importance than that of the gross vehicle weights. Two sets of data on axle weights were available: that for a series of 12 PS trucks provided by Hamblin¹² and the series of 251 PM trucks provided by Buckland & Taylor Ltd.⁹

5.5.1. PS traffic

For the 12 PS trucks, the maximum moment and shear force effect distribution curves were derived using the influence lines for spans of 2–100 m. For short spans, up to about 10 m for moment and 6 m for shear, the statistical distributions resemble very closely that of the heaviest axle rather than that of the gross vehicle weight. For these shear and moment distributions the weighted average $\delta = 0.992$ and $V = 0.105$, while for the heaviest axle $\delta = 0.994$ and $V = 0.109$. Thus the statistical data for the heaviest axle approximate closely that for moment and shear force effects for short spans.

For long spans, the weighted average δ and V for the moment and shear force effect distributions of 0.965 and 0.074 respectively are almost identical with those of the gross vehicle weights of 0.973 and 0.071. Thus the force effect data for long spans can be approximated by those of gross vehicle weights.

Because the distribution curve coefficients (δ and V) for moments and shears were obtained from a small sample of only 12 trucks, these coefficients were both scaled by multiplying by the respective ratios of the coefficients of the gross vehicle weights for the total PS sample of 223 trucks (0.9984 and 0.0571) to those of the gross vehicle weights for the 12 PS trucks (0.973 and 0.071) to give a δ and V for the truck axle force effect event (moment and shear) of 1.018 and 0.0844 respectively.

These event coefficients were then used to derive, for the maximum annual force effect based on 1614 PS trucks per annum, a bias coefficient, δ , of 1.260 and a coefficient of variation, V , of 0.0410.

5.5.2. PM traffic

For the 251 PM trucks (Buckland & Taylor Ltd.⁹), based on the correlation of the bias coefficient, δ , and the coefficient of variation, V , for the heaviest axle weights with the

the force effects of moment and shear force for short spans, the δ and V for the force effects were taken to be the same as those of the heaviest axle weights, i.e., 1.0287 and 0.0072 respectively. These values were again scaled to represent the total population of PM trucks having gross vehicle weight ($\delta = 1.0032$ and $V = 0.0146$) as compared to the Yukon sample of 251 PM trucks with gross vehicle weight ($\delta = 1.0076$ and $V = 0.0064$) to give $\delta = 1.0242$ and $V = 0.0164$ for the force effect event. Finally, the corresponding maximum annual force effect coefficients of $\delta = 1.087$ and $V = 0.0088$ were derived for an annual population of 43 800 trucks, as was used for PM traffic.

5.5.3. NP traffic

No data were available on axle weights for NP (general non-permit) truck traffic. It is expected, based on the studies for PS traffic, that the same correlations between force effects for short spans and axle weights and between force effects for long spans and gross vehicle weights would continue to exist. Furthermore, parallel shifts would be expected to occur in the bias coefficients and coefficients of variation as were established for the other traffic categories. Thus there would be corresponding changes in the load factors for NP traffic on short spans as compared to those for NP traffic on long spans, as was found for PS traffic as tabulated subsequently.

6. Impact

A reanalysis of the data for the positive response region in Table 3 of Billing (1984) gives a mean value of impact, expressed as a fraction of that in the 1988 edition of CAN/CSA-S6, of 0.403 with the extremely large coefficient of variation of 0.983. These data are considered appropriate for NP, PM, and PS traffic on spans greater than about 22 m.

Billing also shows, in his Fig. 9, that the impact fraction decreases with increasing gross vehicle weight, W (kN). An analysis of 16 impact effects for trucks with weights up to 580 kN gives an impact fraction varying inversely as the weight, that is,

$$[31] \quad I = 41.5/W \leq 0.415$$

with a δ of 1.011 and a relatively large coefficient of variation of 0.301. Equation [31] is plotted in Fig. 2 with Billing's data. It is considered conservative to use this impact fraction for PC load effects, even though the latter vehicles may be considerably heavier, because the speeds of PC vehicles are controlled and because of their lengths and number of axles.

The impact data are summarized in Table 5.

FIG. 1. Event and annual exceedance curves for NP traffic.

7. Uncertainty in transverse distribution of live load

For structures that are statically determinate in the transverse direction, the distribution of wheel loads to supporting longitudinal members should be close to exact, giving a δ of 1.00 and a coefficient of variation of 0.00.

Structures are not infrequently analyzed by sophisticated methods of analysis such as finite element methods or grillage analyses, taking into account the flexural and torsional stiffness of the longitudinal members and the flexural stiffness of the slab and diaphragms. Such analyses by Sanders and Elleby (1970) on a series of 11 bridges and by

Dorton *et al.* (1977) on the Conestogo River Bridge were compared with experimental observations. The two sets of data are in close correspondence and give a δ of 1.024 and a V of 0.0914. The δ of 1.024 indicates that the sophisticated analysis slightly underpredicts the experimental value.

Sanders and Elleby also compared the experimental values with those given by the AASHTO code. This comparison results in a δ of 0.924 (the code overestimates the load/girder) with a V of 0.135. The hard metric conversion in CAN/CSA-S6-M88 reduces the δ to 0.893 and increases the V to 0.140.

TABLE 5. Impact fraction

Category	Source	δ	V
NP, PM, PS	Billing (1984)	0.403	0.983
PC	Billing (1984)	1.011	0.301

Hamblin and Moroz have developed an empirical method for distributing PC traffic loads to longitudinal members (Moroz¹⁴), which is not further considered here.

The data are summarized in Table 6.

8. Uncertainty in transverse distribution of dead load

No uncertainty is assumed to be associated with the transverse distribution of the dead load of statically determinate systems. Thus the self-weight of girders and of wet concrete, supported on statically determinate formwork, will fall in this category with $\delta = 1.00$ and $V = 0.00$.

For loads applied after the concrete deck slab has set, when the structure is statically indeterminate, a sophisticated distribution of dead loads may be appropriate. This would apply to barriers, curbs, sidewalks, rails, and wearing surfaces. As no data were available, it has been assumed that $\delta = 1.00$ (all of the dead load will be accounted for) and that V is one half of that for the live load distribution (which is somewhat more uncertain), that is, 0.05.

The data are summarized in Table 7.

9. Resistance data

The statistical data for resistances in Table 8 are based on Kennedy and Baker (1984) for steel and composite members, Fisher *et al.* (1978) for steel connectors, and Mirza and MacGregor (1982) for concrete members. While the data for steel members are directly related to bridge construction, the data for concrete members are more typical of the size of members used in buildings.

10. Load and resistance factors

10.1. Normal loading

The development of consistent load and resistance factors is a multistage process in which regression analyses are performed, simplifications made and re-analyses carried out all with a view to maintaining precision and yet making the system as simple to use as possible. Using the statistical data consisting of bias coefficients, δ , and coefficients of variation, V , for dead, live, and impact loads, for uncertainties in analysis, and for resistances given in Sects. 4–9 and the load effect fractions given in Sect. 3 in [13], an equation for any target value of β is obtained, having as unknowns the resistance factor, ϕ , and the five load factors, α_{D1} , or α_{D2} , α_{D3} , α_{D4} , α_L , and α_I . (Loads D_1 , the dead load of steel girders, and D_2 , the dead load of concrete girders, do not occur simultaneously.)

By setting one of the unknowns at some value, for example, $\phi = 0.94$, as proposed by Kennedy and Baker (1984) for the plastic moment capacity of a steel beam, in [13] and using five sets of load effect fractions from Table 1, five simultaneous equations are obtained to give the values of

FIG. 2. Impact fraction versus gross vehicle weight.

the load factors. Of course, five other sets of load effect fractions would likely give other values of the load factors. Therefore a linear regression analysis was performed on the 41 sets of load effect fraction data to determine the best set of load factors for a particular resistance, type of live load (NP, PS, PM, or PC), corresponding impact, method of lateral load analysis, and β . With the best set of load factors, the variation of β for the 41 sets of load effect fractions was analyzed. These analyses indicated that the variation in β was small with a coefficient of variation of less than about 0.04.

Having therefore established a set of load factors for a particular type of live load, lateral load analysis, and target value of β , resistance factors for all the other resistances are quickly determined. The entire procedure is then repeated for other values of β , other types of live load, and other methods of lateral load analysis, thus covering all possible conditions. (Other analyses can be carried out using statistical data outside that used here to establish a broader range of variations of the load and resistance factors.)

10.2. Resistance factors

To simplify the bridge evaluation procedure, the variation in the resistance factors and dead load factors, as a function of β , was examined for the various types of analyses and categories of traffic. This examination revealed that the variation in resistance factors was relatively minute and therefore *constant resistance factors* were selected for each of the resistances for the range of β appropriate to the function of member or connection behaviour. These are given in Table 9.

Subsequent to the determination of these resistance factors, it was established that the data of Mirza and MacGregor, given in Table 8 for concrete elements, were based on normal distribution curves fitted to the simulation data rather than on log normal distributions as was the case for steel components and as used in calculating the load and resistance factors. When log normal distributions are used, the bias coefficients increase by up to 9% and the coefficients of variation by up to 39%. The two effects are counteractive, but the larger increases in the coefficients of variation predominate and result in reduced resistance factors as given in parentheses in Table 9. The greatest change is for category 15, shear, no stirrups, where the corrected value is 7.6% less. The other reductions are from 0.5% to 3.7%.

¹⁴Moroz, A.W. 1987. Empirical transverse distribution of PC truck loads with girder spacing. Alberta Transportation and Utilities, Edmonton, Alta. Private communication.

TABLE 6. Uncertainty in the transverse distribution of live load

Category	Source	δ	V
Statically determinate in transverse direction	Statics	1.00	0.00
Sophisticated analysis	Sanders and Elleby (1970), Dorton <i>et al.</i> (1977)	1.024	0.0914
Code analysis	Sanders and Elleby (1970)	0.893	0.140

TABLE 7. Uncertainty in the transverse distribution of dead loads

Category	Source	δ	V
Statically determinate	Statics	1.00	0.00
Statically indeterminate	Sophisticated analysis	1.00	0.05

10.3. Dead load factors

The data given in Table 1 shows that 25 of the 41 bridges have steel girders, D_1 , and 16 have concrete girders, D_2 . Furthermore, only 26 of the 41 have cast-in-place decks, that is, a D_3 load effect fraction, and 39 of the 41 have a wearing surface, D_4 .

Linear regression analyses were performed on the sets of data to determine the best set of load factors for a particular resistance (e.g., steel, plastic moment capacity). In carrying out the linear regression analyses, in order to account for the fact that each set of fractions does not contain all the load effect fractions, only those sets of data containing the particular fraction were analyzed. Thus 25 sets were analyzed for steel bridges, 16 for concrete bridges, 26 for cast-in-place decks, and 39 for wearing surfaces. Fractions D_1 , steel girders, and D_2 , concrete girders, are mutually exclusive.

Using the 25 sets of data that have a D_1 fraction, the linear regression analysis gives the best set of load factors to minimize the variation of β for the 25 fractions for one ϕ , type of live load, impact, and method of lateral load analysis. Of all the load factors, only the load factor α_{D_1} is retained from this analysis. This analysis was then repeated for all the types of live load, all methods of lateral load analysis, and all values of β to be considered. An examination of the variation of α_{D_1} with β in this database showed that the variation was essentially linear and therefore a linear regression analysis was performed to arrive at the values given in Table 10 for α_{D_1} .

Subsequently, the procedure was repeated for the 16 load effect fractions ratios that have D_2 as a fraction and the load factor α_{D_2} was determined as a linear function of β . With α_{D_1} and α_{D_2} set as a function of β , the 26 sets of load effect fractions containing D_3 , cast-in-place decks, were analyzed using the above procedure to establish α_{D_3} as a linear function of β . The 39 sets with the load effect fraction D_4 , wearing surface, were then analyzed similarly to establish α_{D_4} as a linear function of β . In each of these analyses, the values of the dead load factors for the preceding dead loads were taken as fixed or set. The values of the dead load factors are given in Table 10.

10.4. Live load and impact factors

Having determined the dead load factors α_{D_1} , α_{D_2} , α_{D_3} , and α_{D_4} as linear functions of β , linear regression analyses were performed on the 41 sets of load effect fraction data,

some of which do not include one or more of D_1 , D_2 , D_3 , and D_4 , to determine the best value of α_L and α_I to minimize the variation in β . These values are given in Table 11. Because the impact fraction for PC traffic is relatively small, it proved advantageous to set the value of α_I for this traffic at the constant value of 1.00.

The values of the resistance factors given in Table 10 and the remaining 15 resistances, using the set values of α_L and α_I for dead loads, live loads, and impact, were then checked to ensure that they had not changed significantly due to the recalculation of the load factors.

It is, of course, recognized that the variation of the load effect fraction influences the load factors for a given resistance, type of load, and method of analysis. Therefore, it is essential that the load effect fraction database be as representative as possible.

10.5. Live load and impact factors, short spans

Live load and impact factors for PS and PM traffic on short spans were determined, based on the statistical data developed for the short span annual maximum force effects, in the same manner as for long spans, taking the resistance factors and dead load factors to be those established for long spans. These values are given in Table 12.

10.6. Counteractive dead load factors

To establish counteractive dead load factors, the resistance factors, live load and impact factors were taken as established for active dead loads. Because of the wide variation in the proportions of load effect fractions when the dead loads are counteractive and because of the relatively small sample size of 15, the counteractive dead load factors tended to vary considerably with the target value of β when using a linear regression analysis.

Based on the above work and using [29], linearly varying values of counteractive dead load factors with β were established such that the target value of β was reached on the average for the different load effect fractions, different resistances, different categories of live loads (NP, PS, PM, PC), and different methods of analysis.

Because, in this approach, the resistance and load factors have been fixed for all other variables, all the deviations for any bridge from the exact value for that bridge appear as a deviation in β from the target value. In spite of this, the mean value of β , examining the extreme values of β for each category of live load and each method of analysis, was 1.013 of the target value with a coefficient of variation varying between 0.070 and 0.156 for the 80 cases investigated. The counteractive dead load factors, varying linearly with β , are given in Table 13.

10.7. Discussion

From Table 10, it is seen that the dead load factor for concrete decks, α_{D_4} , is about the same as those for steel

TABLE 8. Resistance data

Category	Source	δ	V
R ₁ — steel, plastic moment	Kennedy and Baker (1984)	1.133	0.096
R ₂ — steel, yield moment	Kennedy and Baker (1984)	1.221	0.100
R ₃ — steel, inelastic moment	Kennedy and Baker (1984)	1.155	0.084
R ₄ — steel, elastic moment	Kennedy and Baker (1984)	1.090	0.092
R ₅ — steel, composite moment	Kennedy and Baker (1984)	1.098	0.096
R ₆ — steel, compression $\lambda = 0.8$	Kennedy and Baker (1984)	1.058	0.079
R ₇ — steel, bolts, tension	Fisher <i>et al.</i> (1978)	1.200	0.090
R ₈ — steel, bolts, shear	Fisher <i>et al.</i> (1978)	1.250	0.100
R ₉ — steel, welds, fillet	Fisher <i>et al.</i> (1978)	1.316	0.172
R ₁₀ — concrete, cast-in-place beam	Mirza and MacGregor (1982)	1.040	0.085
R ₁₁ — concrete, pretensioned beam	Mirza and MacGregor (1982)	1.05	0.050
R ₁₂ — concrete column, tension failure	Mirza and MacGregor (1982)	0.98	0.08
R ₁₃ — concrete column, compression failure	Mirza and MacGregor (1982)	0.98	0.110
R ₁₄ — concrete, shear with adequate stirrups	Mirza and MacGregor (1982)	1.06	0.140
R ₁₅ — concrete, shear no stirrups	Mirza and MacGregor (1982)	0.95	0.165
R ₁₆ — concrete, overreinforced pretensioned beam	Mirza and MacGregor (1982)	1.05	0.125

TABLE 9. Resistance factors

Category	ϕ	Applicable range of β
Steel		
1. Plastic moment	0.941	1.75–3.25
2. Yield moment	1.007	1.75–3.25
3. Inelastic moment	0.984	2.25–3.75
4. Elastic moment	0.913	2.25–3.75
5. Composite moment	0.912	1.75–3.25
6. Compression member	0.916	2.75–3.75
7. Bolts, tension	1.010	2.75–3.75
8. Bolts, shear	1.029	2.75–3.75
9. Welds, fillet	0.869	2.75–3.75
Concrete		
10. Cast-in-place beam	0.879 (0.869)*	1.75–3.25
11. Pretensioned beam	0.939 (0.934)*	1.75–3.25
12. Column, tension failure	0.839 (0.827)*	2.75–3.75
13. Column, compression failure	0.782 (0.766)*	2.75–3.75
14. Shear, adequate stirrups	0.790 (0.761)*	2.25–3.75
15. Shear, no stirrups	0.630 (0.582)*	2.25–3.75
16. Overreinforced pretensioned beam	0.806 (0.783)*	2.25–3.75

*Corrected values based on log normal distributions.

and concrete girders, α_{D_1} and α_{D_2} . However, the fact that both the bias coefficient and the coefficient of variation for the deck dead load (Table 3) are larger than for steel or concrete girders would be expected to result in a larger load factor for decks. Several checks were therefore made to confirm the validity of the load factors, particularly those for concrete decks.

When several artificial load effect ratios with abnormally high values of the fraction for D_3 were added to the 26 sets of data for real bridges, the value of α_{D_3} increased substantially. This indicates that α_{D_3} , in particular, is sensitive to the load effect fractions used in the analyses.

Next, the coefficients of variation of the estimates of the load factors were determined for eight different cases, consisting of two values of the reliability index of 2.25 and 3.75, two types of load analyses, and two types of traffic. These coefficients were all relatively small, ranging from 2% to 4% for α_{D_1} and α_{D_2} , 4% to 6% for α_{D_3} , and 4% to 9% for

α_{D_4} . The larger values for α_{D_3} show that the confidence limits on α_{D_3} are wider and that the estimate of α_{D_3} is not as precise as the estimate of the other load factors.

As there also appeared to be correlation between the live load and impact, linear regression analyses were performed for the eight cases cited above to determine the effect on the other load factors when the load factor for impact, α_I , was set arbitrarily equal to 1.00. Of course, the live load factor changes to accommodate the fixed α_I . Neither of the load factors, α_{D_1} nor α_{D_2} , was sensitive to this change. On the other hand, the value of α_{D_3} changed by up to 8% in a sporadic manner, sometimes increasing with β and other times decreasing. Again the confidence limits on the estimate of α_{D_3} were wider than those for the other load factors.

The analyses with α_I set equal to 1 also showed that the values of α_L and α_I originally established were not inflated by correlation between the live load and impact load fractions. This implies that the dead load factors considered as

TABLE 10. Dead load factors

Load	$\alpha = f(\beta)$	Load factor, α , for β of							
		2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75
D_1	$0.837 + 0.083\beta$	1.003	1.024	1.045	1.065	1.086	1.107	1.128	1.148
D_2	$0.951 + 0.048\beta$	1.047	1.059	1.071	1.083	1.095	1.107	1.119	1.131
D_3	$0.933 + 0.051\beta$	1.035	1.048	1.061	1.073	1.086	1.099	1.112	1.124
D_4	$0.589 + 0.769\beta$	2.217	2.319	2.512	2.704	2.896	3.088	3.281	3.473

TABLE 11. Live load and impact factors, long spans

		Load and impact factors, α , for β of							
Type of analysis	Load	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75
(a) Non-permit traffic, NP									
Statically determinate	Live		1.406	1.440	1.476	1.515	1.557	1.602	1.649
	Impact		0.786	0.836	0.885	0.933	0.979	1.026	1.072
Sophisticated	Live		1.500	1.545	1.590	1.639	1.692	1.747	1.807
	Impact		0.776	0.822	0.869	0.915	0.959	1.004	1.048
Code	Live		1.380	1.429	1.481	1.536	1.596	1.659	1.726
	Impact		0.734	0.775	0.814	0.853	0.891	0.928	0.965
(b) Permit multiple trip, PM									
Statically determinate	Live		1.094	1.121	1.149	1.178	1.211	1.246	1.282
	Impact		0.721	0.761	0.801	0.840	0.879	0.918	0.956
Sophisticated	Live		1.166	1.199	1.235	1.272	1.313	1.357	1.407
	Impact		0.718	0.757	0.795	0.834	0.870	0.906	0.942
Code	Live		1.075	1.112	1.153	1.195	1.242	1.292	1.347
	Impact		0.674	0.707	0.740	0.771	0.801	0.832	0.861
(c) Permit single trip, PS									
Statically determinate	Live	1.171	1.196	1.224	1.254	1.287	1.322	1.359	1.399
	Impact	0.703	0.749	0.795	0.839	0.882	0.924	0.965	1.006
Sophisticated	Live	1.241	1.274	1.309	1.348	1.389	1.433	1.480	1.530
	Impact	0.699	0.744	0.788	0.830	0.872	0.912	0.951	0.989
Code	Live	1.137	1.174	1.215	1.258	1.305	1.355	1.409	1.466
	Impact	0.661	0.700	0.738	0.774	0.809	0.842	0.874	0.905
(d) Permit controlled, PC									
Statically determinate	Live	1.025	1.052	1.082	1.112	1.145	1.179		
	Impact	1.000	1.000	1.000	1.000	1.000	1.000		
Sophisticated	Live	1.101	1.138	1.176	1.217	1.259	1.304		
	Impact	1.000	1.000	1.000	1.000	1.000	1.000		
Code	Live	1.007	1.047	1.090	1.135	1.183	1.234		
	Impact	1.000	1.000	1.000	1.000	1.000	1.000		

a group are not too low and so establishes more confidence in the entire set of load factors.

Finally, the value of β was determined using the load factors established here for load effect fraction number 37 of Table 1, which has the largest component of load effect due to concrete deck, i.e., 0.425 of the total. The calculated value of 2.95 as compared with the target value of 3.0 indicates that even though the load factor for concrete decks appears low, the safety index is not compromised to any appreciable extent.

It is therefore concluded that the load factors calculated here are certainly appropriate for the 41 sets of load effect fractions on which they are based and also for bridges with ratios within the range of those used here. Of all the load factors, that for concrete decks has the widest limits. Because [13] constrains the factored load effect to equal the factored resistance in the limit, unconservatism only arises when a

particular load effect fraction exceeds the values used in the analysis and when the load factor assigned to it is small.

11. Summary

Data on load effect fractions, the decimal fraction of the various dead and live load components (steel girders, concrete girders, decks, wearing surfaces, truck live loads, and impact), are compiled, based on field surveys, for bridges in Alberta, Saskatchewan, and Ontario. Statistical data in the form of bias coefficients and coefficients of variation are assembled for the various dead and live loads, for impact, for uncertainties in the transverse distribution of live and dead loads, and for resistances. The data for dead loads are based on field surveys in Alberta, Saskatchewan, Ontario, Newfoundland, and the Yukon. The data for live loads are based on field surveys in Alberta, Saskatchewan,

TABLE 12. Live load and impact factors, short spans

		Load and impact factors, α , for β of							
Type of analysis	Load	2.00	2.25	2.50*	2.75*	3.00	3.25*	3.50*	3.75
(a) Permit single trips, PS									
Statically determinate	Live	1.262	1.289	1.319	1.351	1.385	1.422	1.461	1.503
	Impact	0.723	0.771	0.819	0.865	0.912	0.957	1.002	1.045
Sophisticated	Live	1.339	1.374	1.412	1.453	1.497	1.544	1.594	1.647
	Impact	0.716	0.773	0.809	0.854	0.898	0.941	0.983	1.024
Code	Live	1.227	1.267	1.310	1.357	1.407	1.460	1.517	1.578
	Impact	0.677	0.719	0.759	0.797	0.834	0.871	0.905	0.939
(b) Permit multiple trip, PM									
Statically determinate	Live		1.118			1.203			1.311
	Impact		0.728			0.852			0.968
Sophisticated	Live		1.188			1.296			1.430
	Impact		0.725			0.845			0.955
Code	Live		1.096			1.217			1.371
	Impact		0.681			0.784			0.872

*Factors not computed for these values of β in the multiple trip case, but may be estimated by interpolation and with regard to the factors in Table 11.

TABLE 13. Counteractive dead load factors

Load	$\alpha = f(\beta)$	Load factor, α , for β of								
		1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75
D_1	$0.946 - 0.094\beta$	0.781	0.758	0.735	0.711	0.688	0.664	0.641	0.617	0.594
D_2	$0.946 - 0.094\beta$	0.781	0.758	0.735	0.711	0.688	0.664	0.641	0.617	0.594
D_3	$0.782 - 0.112\beta$	0.586	0.558	0.530	0.502	0.474	0.446	0.418	0.390	0.362
D_4	$0.648 - 0.143\beta$	0.398	0.362	0.326	0.291	0.255	0.219	0.183	0.148	0.112

and the Yukon for the four categories of truck traffic of NP, non-permit general highway traffic; PM, permit multiple trip traffic; PS, permit single trip traffic; and PC, permit controlled traffic. Where appropriate, the statistical data for live loads are modified from the event basis to 1 year, the period selected for evaluation.

Using the data on load effect fractions and the statistical data on the loads and analyses, consistent resistance factors, ϕ , and load factors, α , are determined for a broad range of bridge components for the various categories of loads and for a range of values of the reliability index, β , determined by Allen (1992) based on a life-safety criterion as related to the consequences of failure.

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